CHAPTER 7

ENERGY DISSIPATORS

Section I. Basic Considerations

7-l. General. The design of the energy dissipator probably includes more options than any other phase of spillway design. The selection of the type and design details of the dissipator is largely dependent upon the pertinent characteristics of the site, the magnitude of energy to be dissipated, and to a lesser extent upon the duration and frequency of spillway use. Good judgment is imperative to assure that all requirements of the particular project are met. Regardless of the type of dissipator selected, any spillway energy dissipator must operate safely at high discharges for extended periods of time without having to be shut down for emergency repairs. An emergency shutdown of the spillway facility during a large flood could cause overtopping of the dam and/or create unacceptable upstream flooding. The three most common types of energy dissipator used at CE projects are as follows:

a. The stilling basin which employs the hydraulic jump for energy dissipation.

b. The roller bucket which achieves energy dissipation in surface rollers over the bucket and ground rollers downstream of the bucket.

c. The flip bucket which deflects the flow downstream, thereby transferring the energy to a position where impact, turbulence, and resulting erosion will not jeopardize safety of the dam or appurtenant structures.

7-2. Design Discharge. The design discharge for a given spillway energy dissipator must be uniquely determined for each facility and should be dependent upon the damage consequences when the design discharge is exceeded. As a general rule, a spillway energy dissipator should be designed to operate at maximum efficiency and essentially damage-free with discharges at least equal to the magnitude of the standard project flood. The Chief Joseph Dam stilling basin is designed to contain the full spillway design flood (SDF) because failure to do so would compromise the integrity of the project's powerhouse which is located downstream of the basin. The dissipator need not be designed for the spillway design flood if operation with the spillway design flood does not create conditions endangering the dam or causing unacceptable economic damages. Libby Dam is an example where the stilling basin is designed to fully contain the standard project flood while the jump is allowed to entirely sweep out of the basin with a discharge equal to 70 percent of the spillway design flood. A flood that will cause sweepout of this basin would be an extremely remote possibility and would result in damage to the tailrace channel, tailrace channel bridge, and a power transmission tower. However, an economic analysis showed that the cost to dissipate the SDF energy within the stilling basin significantly exceeded the cost to repair and/or replace the damaged features.

7-3. Operation. Optimum energy dissipation will occur when the flow enters the dissipator uniformly. The hydraulic designer is responsible to ensure that project operating schedules are developed to maintain balanced flow operation of a gated, multiple-bay spillway at equal gate openings. The designer must realize, however, that conditions may occur that require unbalanced operation, e.g., development of fish attraction flows, operator error, or emergencies. Such conditions should be considered during evaluation of energy dissipation and stilling basin performance under conditions of nonuniform flow distribution.

Section II. Stilling Basins

7-4. General. The stilling basin employs the hydraulic jump for energy dissipation and is the most effective method of dissipating energy in flow over spillways. The theory of the hydraulic jump is discussed in paragraph 2-13 of this manual. The two basic parameters to be determined for design of a stilling basin are the apron elevation and length. Effective energy dissipation can be attained with a stilling basin having either a horizontal or sloping apron. The use of a sloping or horizontal apron is based solely upon economics in order to provide the least costly basin.

7-5. Horizontal Apron Basin.

a. Apron Length. The optimum stilling basin design would have an apron of sufficient length to confine the entire hydraulic jump. The jump length is a function of entering Froude number F_1, and entering depth, d_1. The approximate length of a hydraulic jump on a flat floor is $3.5d_1F_1^{1.5}$ for F_1 less than five and $8.0d_1F_1$ for F_1 greater than five (item 40). However, a basin of such length is normally not cost-effective. Appurtenances such as baffle blocks and end sills on the apron can be used to decrease the length of the jump without compromising the efficiency of energy dissipation. A limited review of stilling basins for high and low head spillway structures has shown that a stilling basin length can be reasonably defined by the equation:

$$L_b = Kd_1F_1^{1.5} \qquad (7\text{-}1)$$

where K is the stilling basin length coefficient from Table 7-1. Equation 7-1 is considered applicable in the range of Froude numbers, F_1, between 2 and 20, and will provide a basin length that is adequate for feasibility level designs and the basic basin length necessary to proceed into model verification. The coefficient K in equation 7-1 has been found to vary between 1.4 and 2.0 dependent upon the use of baffles and end sill. This coefficient is also dependent on the basin use, such as single gate or other unbalanced spillway flow conditions commonly found with low head navigation structures. Table 7-1 gives values for various conditions.

b. Apron Elevation. The optimum design for a stilling basin without baffles would have an apron elevation such that the jump curve defining the required d_2 depth would superimpose on the tailwater curve for the full range of discharge. However, only in extremely rare circumstances will site and hydraulic conditions coexist that result in the jump curve superimposing

TABLE 7-l

Values of K for Various Types of Stilling Basins

Type of Stilling Basin	K	Remarks
Stilling basin with a vertical, stepped, or sloping end sill and one or two rows of baffles	1.4	Items 41, 53, 54, 67, and 72 Suggested upper limit of F_1 is approximately eight
Stilling basin with a vertical, stepped, or sloping end sill only	1.7	Items 38, 58, 60, and 62
Stilling basin for low head broad-crested weir navigation dam spillways with one or two rows of baffles and a sloping end sill	2.0	See EM 1110-2-1605

on the tailwater curve. Experience indicates that if less than optimum energy dissipation can be tolerated, satisfactory performance can be maintained with a stilling basin that includes baffles and end sill when the apron elevation is set at full d_2 depth at the stilling basin design discharge and not less than $0.85d_2$ depth at the spillway design flood. If optimum energy dissipation is required, the basin apron should be set to provide for full d_2 depth with the spillway design flood. Excessive tailwater tends to hold the spillway jet against the apron resulting in high velocity flow exiting over the end sill which may cause damage in the exit channel. Baffles located on the apron will deflect the jet upward through the tailwater to assist in energy dissipation even when tailwater depth is excessive. When determining the apron elevation, the hydraulic designer must evaluate the potential for tailwater changes resulting from downstream channel aggradation or degradation during the life of the project and design the basin accordingly.

7-6. Sloping Aprons. Depending on site foundation conditions, some degree of economy may be realized if the stilling basin is designed with a downstream sloping apron rather than horizontal apron. The hydraulic jump is allowed to form on a portion, or all, of the sloping apron. Plates 7-l and 7-2, which were developed from tests by USBR (item 40), can be used to determine the jump length and tailwater depth required to evaluate the hydraulic jump on aprons of various slopes. In design of a basin, either with a continuous or a noncontinuous slope, baffles and an end sill should be considered. The basin apron can be considered horizontal when the slope is flatter than 1V:6H.

7-7. Baffles.

a. General. Baffles are frequently used to aid in formation of the hydraulic jump. Their use can significantly reduce the length of the jump, decrease the required d_2 depth for a given discharge condition, and provide stability to the jump. Baffle location, shape, size, and spacing are the important parameters to be considered in design of a baffle-aided stilling basin. Cavitation damage on baffles and surrounding surfaces will occur when baffles are used in conjunction with high Froude number flow. The stilling basin design discharge, Froude number and the expected frequency and duration of use are major factors that must be included in the decision to include baffle blocks on a stilling basin apron. The USBR (item 40) recommends the upper Froude number be limited to about 5.8 for a baffled basin when the basin is to be used frequently for such structures as canals, outlet works, and small spillways. Baffles have been used in the Chief Joseph Dam stilling basin (item 53) which has a design discharge Froude number of about five and is designed for frequent use over long-duration flood events. The baffles at Chief Joseph Dam have experienced significant cavitation damage. Green Peter Dam (item 54) has two rows of baffles with a relatively high design discharge Froude number of 8.5. The spillway of Green Peter Dam is expected to be used quite infrequently and for relatively short duration events; however, this stilling basin also provides energy dissipation for flow through the sluices which operate frequently for relatively short periods of time.

b. Shape. The standard CE baffle (Plate 7-3) with a rectangular upstream face and sloping downstream face is the preferred shape. Although a 6-inch bevel on all edges is acceptable, streamlined baffles are not recommended. Streamlining the baffles does not provide as effective energy dissipation as the standard baffle, and contrary to belief, is more likely to cause cavitation damage to the stilling basin floor and to the baffle.

c. Location. The first (upstream) row of baffles plays a dominant role in establishing the type of hydraulic action that the stilling basin will display. Baffles located too far downstream reduce the basin's effective length, while baffles located too far upstream will result in spray originating from the baffle faces. Tests accomplished at WES (item 35) indicate that the optimum location of the baffles is a function of entering Froude number. Data in Plate 7-4a define the location of the upstream face of the first row of baffles. Model studies for which qualitative scour tests were conducted indicate that the second row of baffles assists in decreasing scour downstream from the stilling basin. A second row of baffles should be considered where downstream channel scour is expected to be a problem. When a second row of baffles is used, the upstream face of this row should be located about two and one-half baffle heights downstream from the upstream face of the first row and staggered with respect to baffles in the first row. Minimum spacing between the basin sidewall and a baffle is that required for forming purposes, with the maximum spacing being about one-half baffle width.

d. Size. The baffle height is a function of the entering Froude number as shown in Plate 7-4b. With Froude numbers less than 4.6, the baffle height should be $d_2/6$. The baffle width is essentially equal to the height although any reasonable width less than the height is satisfactory.

7-8. End Sills. An end sill is commonly used as the terminal wall of a stilling basin and forms a step or rise to channel bed elevation. The end sill deflects the higher velocity filaments which exist near the basin apron away from the channel bed. Results of qualitative scour tests with stilling basins containing baffles indicate that minimum exit channel scour results when the end sill has a height of $d_1/2$ or $d_2/12$, whichever is lower.

Higher end sills result in deeper scour near the end sill while low sills result in longer and deeper scour holes. The shape of the end sill does not affect its performance. A 1V on 1H sloping face end sill has the advantage of minimizing the potential for debris to be trapped in the stilling basin.

7-9. Sidewalls. Vertical stilling basin walls are preferred over battered walls because of unacceptable eddy conditions which occur with battered walls. When battered walls are required, the width at midheight of the stilling basin should equal the spillway width to minimize expansion and contraction of flow at the design discharge. Sidewalls should extend at least to maximum tailwater elevation, since return flow over stilling basin walls may create unsatisfactory basin performance, such as drowning of the jump, excessive turbulence, and localized scour downstream from the basin. Model studies are recommended when stilling basin design includes battered or low sidewalls. Computation of hydrodynamic forces acting on stilling basin sidewalls is discussed in paragraph 2-13.

7-10. Wing Walls. A design with free-standing sidewalls is preferable to one incorporating wing walls. Wing walls tend to reflect waves, resulting in a more severe attack on the exit channel side slope than that resulting when the basin sidewalls are terminated at the end sill. When wing walls are required for structural reasons, a wall rotated 90 degrees from the sidewall is preferable to other alignments.

7-11. Exit Channel.

a. General. Except in some unusual conditions, an exit channel is required to transition between the stilling basin and the main channel of the river. Since dissipation of the entire spillway discharge energy within the stilling basin is not normally accomplished, enlarging the channel width immediately downstream from the basin will assist in dissipating the residual energy. Due to the erosive nature of the highly turbulent flow exiting from a stilling basin, protection of the exit channel bed and side slopes is usually required to prevent channel scour and potential undermining of the stilling basin.

b. Size and Shape. The toe of the exit channel should be offset away from the sidewall a distance of $0.15d_2$ or at least five feet. The invert elevation of the exit channel immediately downstream of the end sill should be set 0.25-0.5 times the 100 percent diameter of stone, d_{100} , used for channel protection below the top of the end sill. The setting of the channel invert lower than the end sill is beneficial in reducing the hydrodynamic lift and drag on the stones. Mild invert slopes are recommended to transition the exit channel to the river bottom. At Libby Dam, the originally designed 1V on 6H sloping runout proved to be unstable during prototype operation and was subsequently modified to 1V on 10H. In some instances, sloping depressions or level areas immediately downstream from the end sill have been used to minimize potential for material to migrate down the runout slope and enter the stilling basin. Exit channel designs which abruptly contract the flow downstream from the basin tend to induce lateral eddies and should be avoided.

c. Protection. Unless sited in high-quality rock, the exit channel will require protection to prevent scour and potential damage to the stilling basin. Flow leaving a stilling basin is highly turbulent and as such has a larger erosive force than that due to similar velocities in a low turbulence area. Guidance for design of rock protection adjacent to stilling basins is given in HDC 712-1. Protection based upon this guidance should extend a distance of $10d_2$ downstream from the stilling basin end sill and transition to the natural channel using gradually varying gradations as necessary to prevent major changes in adjacent rock sizes. The designer should be aware that inadequately sized rock or spalls could potentially be transported back into the stilling basin and cause significant damage. Model studies may be necessary to confirm design of the exit channel protection measures.

7-12. Abrasion and Cavitation. Stilling basin damage can occur as a result of abrasion, cavitation, or a combination of both. As discussed in Chapter 2, cavitation is possible wherever boundary irregularities cause a separation of flow with resultant localized pressure drops. In stilling basins, locations where irregularities may exist are at and around baffles, at misaligned joints, and at other irregularities. Cavitation damage is distinguished by its ragged, angular appearance. Abrasion damage, on the other hand, has a smooth and rounded appearance and can be attributed to rock and debris moving through or being trapped in the basin. Depressions which are initially caused by abrasion can form boundary irregularities sufficient to initiate cavitation damage. Rock, gravel, scrap metal, and other hard material may find their way into the energy dissipator by various means. Rock may be carried into a stilling basin by diversion configurations and project operation during the project's construction or by eddies transporting debris in from the downstream channel. In some cases, contractors may fail to clean out all hard material after construction, or rocks may be thrown into a basin by the public. Unbalanced gate operation in a multibay, gated spillway can create strong eddy conditions which draw material from the downstream channel into the basin. Major stilling basin damage requiring dewatering and costly repairs occurred at Libby and Dworshak Dams (item 47) as a result of abrasion following three years of operation (Figure 7-1). Practical measures which can be taken during design, construction, and operation of a project to reduce the possibility of damage to stilling basins are as follows:

a. Use wider exit channels with mild upward sloping runouts to transition from the basin apron to the river channel.

b. Specify close tolerances at construction joints and ensure that construction inspection enforces those tolerances.

c. Avoid baffles in high Froude number basins and never join baffles to basin sidewalls.

Figure 7-1. Damage to Dworshak Dam stilling basin

d. Require that all channel excavations and erosion protection measures downstream and adjacent to basins be complete prior to operation of the basin.

e. Provide barriers around and above basins to prevent construction material from falling into the basin.

f. Plan diversions to reduce potential for depositing material adjacent to basins.

g. Require inspections and cleanup of basins at end of construction.

h. Require basins to be operated with balanced flow conditions.

i. Require regular monitoring of basins.

j. When material is known to be in the basin, immediately remove the material either by flushing with a uniform distribution of water, if possible, or by shutdown and removal by other means.

Hydraulic models may be used to plan and design diversions and operation during construction, to determine flow conditions substantial enough to flush material out of a basin, and to evaluate the effect of nonuniform flow distribution on eddy conditions in basins.

Section III. Roller Buckets

7-13. General. A roller bucket energy dissipator consists of a circular arc bucket tangent to the spillway face terminating with an upward slope. This geometry when located at an appropriate depth below tailwater will produce hydraulic conditions consisting of a back roller having a horizontal axis above the bucket and a surge immediately downstream from the bucket. Solid and slotted buckets have been used successfully. The boundary geometry of a solid roller bucket is similar to that for a flip bucket except that the roller bucket is located well below the tailwater elevation. The geometry of a slotted bucket is variable; however, it is similar to the solid bucket except for the addition of dentates on the downstream quadrant and a downstream apron. A roller bucket can be used where excessive tailwater depths exist either from hydraulic characteristics of the river channel or foundation conditions that require siting an energy dissipator well below the depth necessary for adequate hydraulic jump energy dissipation. For adequate energy dissipation to occur with a roller bucket, the tailwater depth must be within defined limits. These limits are dependent upon the inflow energy and the bucket radius. Insufficient tailwater depth will result in the flow sweeping out of the bucket and forming a jet, typical of a flip bucket. A more undesirable condition can occur just prior to sweepout when an instability develops which could result in excessive erosion and undesirable wave conditions in the tailrace and downstream channel. Excessive tailwater depth will cause the flow to dive from the bucket lip resulting in the development of a roller and surging downstream from the bucket. This action will cause erosion and movement of large volumes of bed material resulting in hydraulic instabilities, inadequate energy dissipation, and bucket erosion. Because the bucket is located immediately adjacent to the toe of the spillway, the roller bucket should be designed to efficiently dissipate the energy of the spillway design discharge to ensure against compromising the integrity of the dam structure proper. Appendix F contains an example problem for the design of a roller bucket.

7-14. Bucket Depth and Radius. The hydraulic design of the roller bucket is derived strictly from empirical data, the majority of which is from model studies (item 35). The minimum radius for a roller bucket, r_{min} , is defined as

$$r_{min} = \frac{5.19\left(d_1 + \frac{V_1^2}{2g}\right)}{F_1^{1.64}} \tag{7-2}$$

where

d_1 = depth of flow entering the bucket, feet
V_1 = velocity of flow entering the bucket, ft/sec
F_1 = Froude number of the entering flow

The bucket invert elevation limits, maximum tailwater depth h_2max , and minimum tailwater depth h_2min , are related to the bucket radius r , F_1 , d_1 , and the specific energy of the entering flow $d_1 + V_1^2/2g$. These relationships are provided in Plates 7-5 and 7-6. The roller height h_b and the surge height h_s are related to the difference in reservoir and bucket invert elevations h_1 , the tailwater height h_2 , and the parameter $(q \times 10^3)/(g^{1/2} \cdot h^{3/2})$ as shown in Plates 7-7 and 7-8. The important characteristics which must be evaluated in design of the roller bucket include the minimum tailwater depth which does not result in bucket sweepout, the maximum tailwater depth at which diving of the jet does not occur, and the maximum surge height downstream of the bucket and the height of the back roller above the bucket. Hydraulic model tests to verify the design of roller buckets are recommended under the following conditions:

a. Sustained operation near the limiting conditions is expected.

b. Discharges exceed 500 ft^3/sec per foot of width.

c. Velocities entering the bucket exceed 75 ft/sec.

d. Eddies appear possible.

e. Waves in the channel downstream from the structure would be a problem.

7-15. Slotted Buckets. A disadvantage of the solid roller bucket is that the downstream surge can move loose material from the channel bed back into the bucket where the action of the back roller can result in serious abrasion damage to the bucket surfaces. For this reason, USBR (item 40) developed a slotted bucket design which reduces the possibility of extraneous material being drawn back into the back roller. The slotted bucket also exhibits better self-cleaning properties. The slotted bucket disperses and distributes flow into the downstream surge over a greater depth resulting in less violent flow concentrations than does the solid bucket (item 34). The slotted bucket developed by USBR consists of upward rounded teeth with vertical sides and a rounded top. This slotted bucket configuration also includes a 16-degree upward-sloping, 20-foot-long apron downstream from the teeth. Model studies of the Little Goose Dam spillway (item 23) were made to develop a design having more easily constructed, plane surface teeth rather than the curved surface design developed by USBR. The Little Goose Dam studies resulted in a design (Plate 7-9) which consisted of teeth trapezoidal-shaped in cross section with an apron configuration downstream from the teeth identical to that of the USBR design. In addition to the less complicated geometrical shape, the Little Goose bucket teeth exhibited more acceptable pressures than the curved-shaped design.

7-16. Exit Channel. Because of the roller bucket's tendency to move loose material from the downstream channel into the bucket itself, design of the exit channel is relatively critical to acceptable performance of the structure. As previously discussed for the hydraulic jump stilling basin, gently sloped well-protected runout slopes should be used to transition from the roller bucket to the river channel. Roller bucket surging will result in the propagation of waves throughout the tailrace and in the downstream channel. The effect of these conditions on the river banks and other structures must be considered. Hydraulic models are necessary to evaluate, at least qualitatively, the performance of the exit channel.

Section IV. Flip Buckets

7-17. General. The flip bucket itself is not an energy dissipator; however, it is an integral part of an energy dissipation system. The purpose of the flip bucket is to direct high-velocity flow (the jet) well away from the dam, powerhouse, spillway, and/or other appurtenances. A small amount of energy is dissipated by friction through the bucket. During the jet's trajectory to its impact location, extremely turbulent flow exists and the jet spreads and frays. The extreme turbulence of the jet entrains a large volume of air. A portion of the jet's energy is dissipated by the interaction of the water and the air boundary resulting in considerable spray. The effect of heavy spray on adjacent structures, especially in cold regions, should be considered. The impact of the jet and the interaction of the turbulent flow and the boundary at the impact area account for the major portion of energy dissipation. The impact will almost certainly cause adjustment to the riverbed even if the bed material is rock. For this reason, use of a flip bucket should be considered only where bed scour caused by the impact of the water jet cannot endanger the dam, power plant, or other structures (including the flip bucket itself) or cause unacceptable environmental damage. Where the flip bucket can be appropriately used, it offers an attractive economical alternative to a stilling basin or roller bucket structure; however, the flip bucket includes more uncertainties as to adequacy than do stilling basins or roller buckets. The parameters of prime importance to the hydraulic designer are the bucket geometry, pressures acting on the bucket boundaries, and the jet trajectory characteristics. Flip bucket design is based on empiricism essentially derived from model studies. For this reason, any deviations from the flip bucket design parameters and guidelines discussed in this manual should be verified by hydraulic model studies. Appendix F contains an example problem for the design of a flip bucket.

7-18. Bucket Geometry.

a. General. The geometric parameters required for design of a flip bucket include the bucket radius, r, the minimum height of the bucket lip, h_{min}, the trajectory angle at the end of the bucket, θ, the bucket invert elevation, and the planimetric alignment of the bucket. The parameters r, h_{min} and θ are closely related and may require trial-and-error adjustment in order to obtain a satisfactory design. The planimetric alignment can be developed to direct the location of the jet impact area. Figure 7-2 depicts the various terms used for the flip bucket design process.

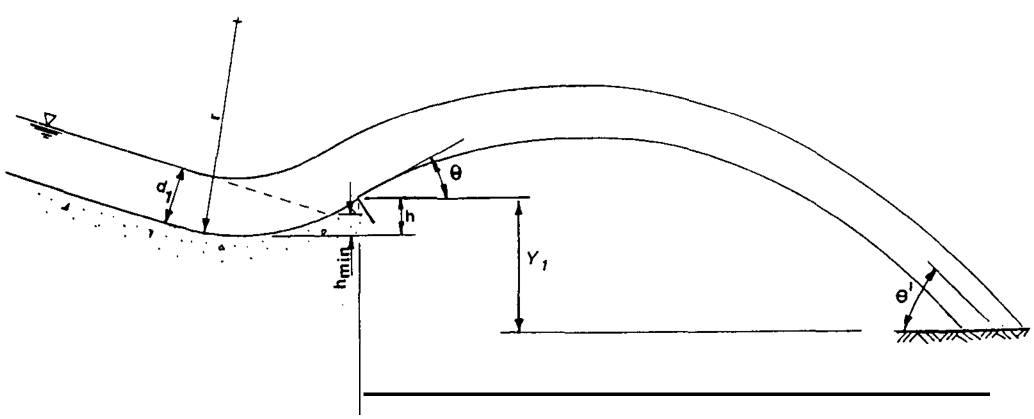

Figure 7-2. Parameters used in the design of a flip bucket

b. Radius. The minimum radius, r_{min}, is a function of the allowable theoretical unit load on the bucket invert P_T, the velocity of flow, V_1, and the depth of flow, d_1, entering the bucket defined as

$$r_{min} = \frac{\rho V_1^2 d_1}{P_T - \gamma d_1} \tag{7-3}$$

As a general design guide, previous experience suggests that a bucket radius at least equal to four times the maximum flow depth will turn most of the water before it leaves the bucket.

c. Minimum Height. The height of the bucket lip must be sufficient to prevent the water from merely overriding the bucket lip in lieu of being turned and flipped out of the bucket. To effectively turn the flow, the bucket height must be at least high enough to intersect the forward-projected slope of the water surface at the point of curvature of the spillway and the bucket curve. The minimum bucket height described by equation 7-4 will ensure that the flow will follow the bucket curve and not override the downstream lip.

$$h_{min} = r - r \cos(\phi - \tan^{-1} S) \tag{7-4}$$

where

$$\phi = \tan^{-1}\left\{\frac{\left[d_1(2r - d_1)\right]^{1/2}}{r - d_1}\right\}$$

describes the minimum deflection angle and S is the slope of the spillway chute adjacent to the bucket.

When $\phi > \tan^{-1} S$, the minimum height of the bucket becomes zero. The height of the bucket is then defined by the required trajectory angle θ. A trial-and-error adjustment of the bucket radius and/or bucket flip angle may be necessary to meet or exceed the minimum bucket height as defined in equation 7-4.

d. Trajectory Angle. The trajectory angle is the angle the bucket lip makes with respect to the horizontal. The trajectory angle is a factor in determining the length of the jet trajectory distance and the general hydraulic characteristics in the impact area. Steeper angles increase the trajectory length and provide better dissipation than flatter angles as they cause the jet to impact in a more vertical direction with less violent side eddies. A 45-degree flip angle will result in the maximum trajectory distance. The required height of the bucket lip, h, above the bucket invert necessary to satisfy the desired trajectory angle θ can be determined by the following equation:

$$h = r - r \cos\theta \tag{7-5}$$

e. Bucket Elevation. For optimum performance, the flip bucket cannot operate under submerged conditions. Depending on the shape of the tailwater curve, raising of the bucket invert elevation or the lip of the bucket may be required. In evaluating tailwater conditions, the designer should consider that the ejector action of the jet as it exits the bucket may tend to cause a drawdown in the tailwater elevation depending on downstream channel geometry. Such drawdown may adversely impact the operation of adjacent structures such as powerhouses, etc. The amount of drawdown which may occur with any given design can best be determined from hydraulic models. For preliminary design purposes, a method of estimating drawdown can be found in item 40.

f. Bucket Termination. The bucket should terminate with a 90-degree cut from the bucket lip, and the sidewalls should terminate at the lip to allow sufficient air to be drawn below the point of the trajectory separation from the bucket lip. Failure to allow sufficient air to the underside of the jet will cause jet flutter with resultant pressure fluctuations and possible cavitation damage. The original design of the flip buckets on the Wynoochee Dam outlets (item 55) terminated in a 20-degree cut which resulted in cavitation damage to the concrete surfaces downstream from the lip. Extending the bucket length to allow a 90-degree termination cut has eliminated this damage.

g. Alignment. The flip bucket can be aligned to direct the trajectory impact to a preselected location by curving or adding appurtenances to the bucket. An example of such a directional design is the spillway for the East Branch Reservoir spillway (item 63). Model studies are required to confirm the final design of a directional flip bucket. A bucket alignment which spreads the flow at the impact area across as much of the river channel as possible minimizes riverbed adjustment and return flow from the downstream tailwater.

7-19. Discharge Considerations. Flip buckets perform best when the entering flow is at high velocity and low unit discharge as such conditions result in considerable fraying of the jet by air resistance. Moderately high unit discharges, however, should not be a problem if downstream channel adjustment is not of prime consideration. The flip buckets at Wynoochee Dam (item 55) have operated satisfactorily for extended periods with unit discharges of approximately 350 ft^3/sec/ft. The Applegate Dam spillway flip bucket was developed through model studies (item 61) and is designed for a unit discharge of 850 ft^3/sec/ft. Flip buckets exist where design unit discharges are well in excess of 1,000 ft^3/sec/ft; these designs are extremely critical with respect to cavitation damage due to the extremely high velocities, deep flow depths, and subatmospheric pressures which exist. Model studies are recommended for flip buckets designed with unit discharges in excess of 250 ft^3/sec/ft.

7-20. Trajectory Distance. The jet trajectory distance is dependent upon the velocity of flow entering the flip bucket, the trajectory angle, and the vertical distance from the bucket lip to the impact area. The trajectory distance, X_H, which is the horizontal distance from the bucket lip to the impact location, is determined by the equation:

$$X_H = h_e \sin 2\theta + 2 \cos \theta \left[h_e (h_e \sin^2 \theta + Y_1) \right]^{1/2} \tag{7-6}$$

where

h_e = velocity head at the bucket lip, feet
Y_1 = vertical distance below the bucket lip to the impact area, feet

When the Y_1 value is equal to zero, then equation 7-6 reduces to:

$$X_H = \frac{V^2}{2g} \sin 2\theta \tag{7-7}$$

The angle at which the jet strikes the impact location, θ', is described by the following equation:

$$\theta' = \tan^{-1} \left[\sec \theta \left(\sin^2 \theta + \frac{Y_1}{h_e} \right)^{1/2} \right] \tag{7-8}$$

Equation 7-8 reduces to $\theta' = \theta$ when Y_1 is equal to zero. Trajectory lengths based on equations 7-6 and 7-7 have been simulated reasonably well in hydraulic models. Prototype trajectories are somewhat shorter and have steeper impact angles than the model or theoretical jet due to the greater air resistance encountered in the prototype.

7-21. Bucket Pressures. Pressures on the invert of the bucket vary throughout the curve and are influenced by the curve radius, the total head on the invert, and the unit discharge. A WES study (item 71) indicated that, for relatively high dams, bucket pressures could be expressed as:

$$h_p = f \left(\frac{q}{r(2gH_T)^{1/2}} , \frac{\alpha}{\alpha_T} \right) \tag{7-9}$$

where

h_p = pressure head against boundary, feet
H_T = total head (point to energy gradient), feet
α = angle of rotation from beginning of curve, degrees
α_T = total deflection angle, degrees

The term α/α_T defines the relative position along the curve. The pressure distribution throughout the length of the flip bucket can be estimated using the data provided in Plate 7-10. This curve has been developed from model data although some prototype data at a small discharge has been included. This curve is essentially the same as HDC Chart 112-7 plotted in a different form. The term $q/\left[r(2gH_T)^{1/2}\right]$ has been replaced by $d_1^{1/2}/r$ for the usual case where d_1 is small when compared to $V^2/2g$. See HDC Chart 112-7 for a more detailed discussion on the data used for Plate 7-10.

7-22. Exit Channel. Optimum performance will occur when the jet trajectory at impact spreads approximately across the entire width of the river channel. Unless the jet impact area is located in extremely durable rock, a scour hole can be expected to occur at the impact point. The material scoured in development of the hole will be deposited downstream where it may adversely impact satisfactory operation of the flip bucket. A preformed scour hole at the impact area can be used to minimize deposition in the downstream channel. Violent wave action can be expected in the impact area, and wave and high-velocity turbulence will likely extend laterally and downstream from the impact. These conditions can lead to streambank damage unless the banks are adequately protected. A model study is recommended to qualitatively evaluate the extent of bed scour and hydraulic conditions existing with operation of a flip bucket.

7-23. Miscellaneous.

a. Drainage. The bucket must be adequately drained to prevent water impoundment in the bucket. Due to potential for cavitation damage, floor drains should be avoided and the bucket should be drained laterally through the sidewalls.

b. Low Flow Operation. At low flows, water may pond in the bucket and spill over the lip. Erosion may be caused by these low flows which do not flip and should be considered in the design. A concrete slab, cutoff wall, or large stone may be needed at the toe of the structure to protect the structure from undermining. A double-flip bucket design was developed for the Applegate Dam spillway (item 61) to prevent damage which would result with operation of low, nonflipping discharges.

Section V. Specialized Energy Dissipators

7-24. Impact Basin. An impact hydraulic jump-type energy dissipator was developed by Blaisdell (item 9) for small drainage structures. The USBR uses a similar dissipator which they designate as a Type III Basin (item 40). Tests at WES on the Rend Lake (item 17) and Oakley (items 31 and 73) projects showed this type basin to be very effective in the Froude number range of 2.5 to 4.5. Preferred dimensions of the basin and its elements for use in this range of Froude numbers are given in Plate 7-11. This type dissipator is not recommended where velocity entering the basin exceeds 60 ft/sec as the chute blocks would be subject to damage by cavitation. An apron length equal to at least $3d_2$ for flows up to the standard project flood, and $2d_2$ for the spillway design flood is considered adequate. The basin elevation should provide a depth on the apron of d_2 for the standard project flood and at least $0.85d_2$ for the spillway design flood.

7-25. Baffled Chute. The baffled chute spillway relies upon multiple rows of baffles to aid in dissemination of energy flowing down a spillway chute. The USBR (item 40) has developed a set of design guidance which can be used in preliminary design of such a structure. Large baffled chute spillways have been used on the Tennessee-Tombigbee Waterway divide cut to convey the flow of streams intercepted by the canal down the cut slope into the canal (item 3). Model studies are recommended for design verification when the design discharge exceeds 50 ft^3/sec and/or the slope is steeper than 1V on 2H. A baffled chute design was developed via model study (item 59) for the proposed Libby Reregulating Dam which was effective not only in energy dissipation, but also in aerating the flow and reducing nitrogen supersaturation. The specially designed baffle (Plate 7-12) for this structure exhibited good aeration characteristics for discharges up to 180 ft^3/sec/ft and adequate energy dissipation for discharges as high as 900 ft^3/sec/ft.